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Development and Calibration of a Two-Dimensional Hydrodynamic Model of the Tanana River near Tok, Alaska

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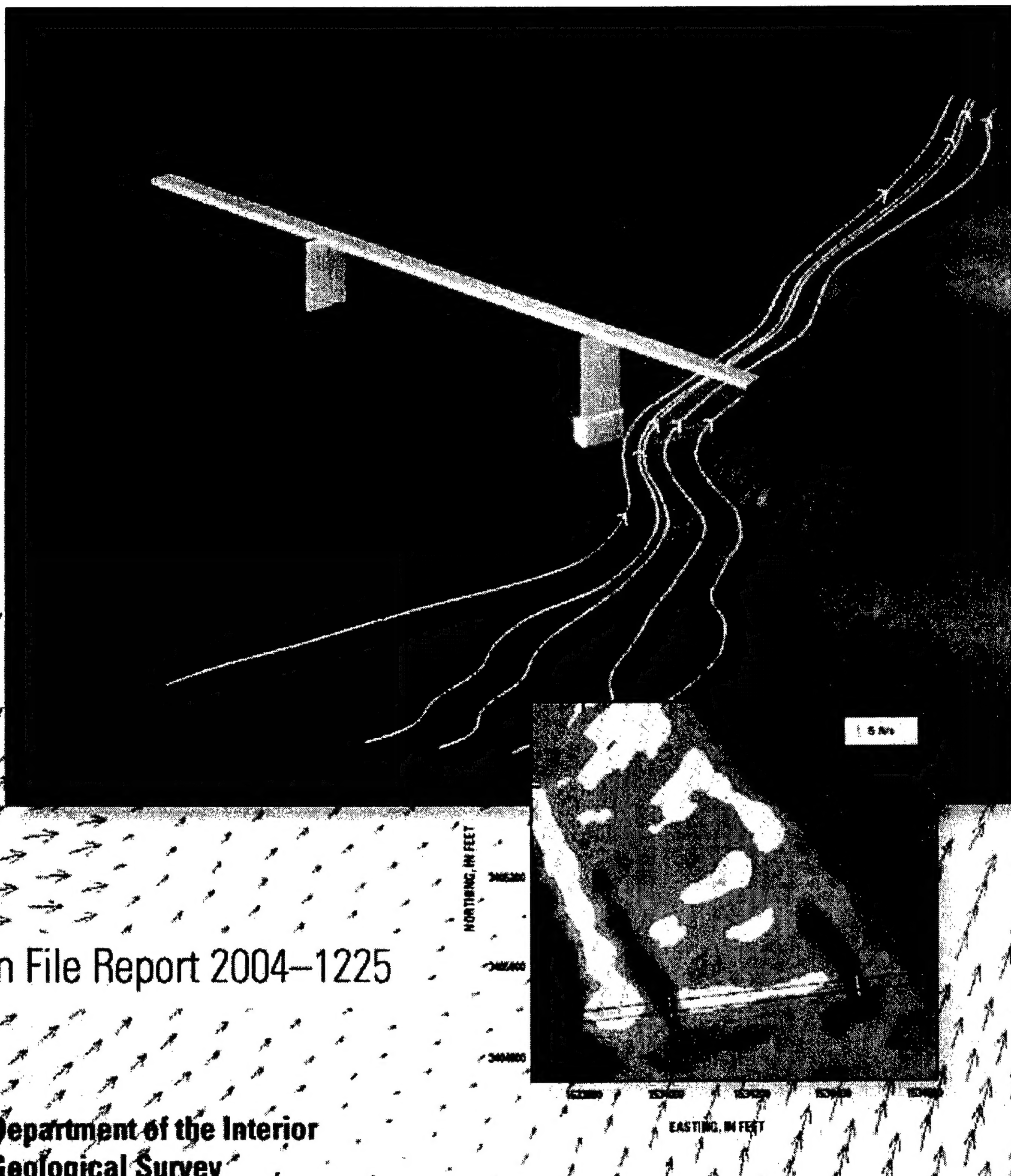

Open File Report 2004–1225

U.S. Department of the Interior
U.S. Geological Survey

Development and Calibration of a Two-Dimensional Hydrodynamic Model of the Tanana River near Tok, Alaska

By Jeffrey S. Conaway and Edward H. Moran

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Suggested citation:
Conaway, J.S., and Moran, E.H., 2004, Development and calibration of a two-dimensional hydrodynamic model of the Tanana River near Tok, Alaska: U.S. Geological Survey Open-File Report 2004-1225, 13 p.

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CONVERSION FACTORS

Multiply	By	To obtain
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
foot (ft)	0.3048	meter
foot per second (ft/s)	0.3048	meter per second
foot squared per day (ft^2/d)	0.09290	meter squared per day
mile (mi)	1.609	kilometer
square mile (mi^2)	12.590	square kilometer

DATUMS

Vertical coordinate information is referenced to the North American Vertical Datum of 1988 (NAVD 88).

Horizontal coordinate information is referenced to the North American Datum of 1983 (NAD 83).

Development and Calibration of a Two-Dimensional Hydrodynamic Model of the Tanana River near Tok, Alaska

By Jeffrey S. Conaway and Edward H. Moran

Abstract

Bathymetric and hydraulic data were collected by the U.S. Geological Survey on the Tanana River in proximity to Alaska Department of Transportation and Public Facilities' bridge number 505 at mile 80.5 of the Alaska Highway. Data were collected from August 7-9, 2002, over an approximate 5,000-foot reach of the river. These data were combined with topographic data provided by Alaska Department of Transportation and Public Facilities to generate a two-dimensional hydrodynamic model.

The hydrodynamic model was calibrated with water-surface elevations, flow velocities, and flow directions collected at a discharge of 25,600 cubic feet per second. The calibrated model was then used for a simulation of the 100-year recurrence interval discharge of 51,900 cubic feet per second. The existing bridge piers were removed from the model geometry in a second simulation to model the hydraulic conditions in the channel without the piers' influence. The water-surface elevations, flow velocities, and flow directions from these simulations can be used to evaluate the influence of the piers on flow hydraulics and will assist the Alaska Department of Transportation and Public Facilities in the design of a replacement bridge.

Introduction

The Alaska Highway, which is Alaska's main land arterial with Canada and the contiguous United States, crosses the Tanana River near Tok, Alaska, via a bridge that is nearly 60 years old (fig. 1). Age, a narrow bridge deck, and the potential for streambed scour at the bridge have led to the Alaska Department of Transportation and Public Facilities' (ADOT&PF) decision to replace the structure. Potential locations for a new bridge are at the existing crossing and immediately downstream. The final choice for location will be based on river hydraulics, right of ways, and feasibility of road realignment.

Bridge designers are required to engineer new structures to accommodate 100-year recurrence interval flood discharges and to withstand the associated streambed scour. An initial step in the design is selecting a location that will have low susceptibility to general scour over a range of hydraulic conditions. If properly designed the location, dimensions, and alignment of bridge piers can reduce the potential for local streambed scour at the piers. In Alaska, hydraulic data are sparse and there is an even greater paucity of data for flood discharges. Flow fields observed at low stages can be either dampened or intensified by high flows and are not applicable for design purposes. An extensive bathymetric data set and properly calibrated hydrodynamic model can provide bridge designers with the necessary hydraulic information needed to locate and design a sound structure. Critical data for design of the replacement structure for bridge 505 are predicted water-surface elevations, velocity magnitudes, and flow directions for the 100-year recurrence interval flood discharge.

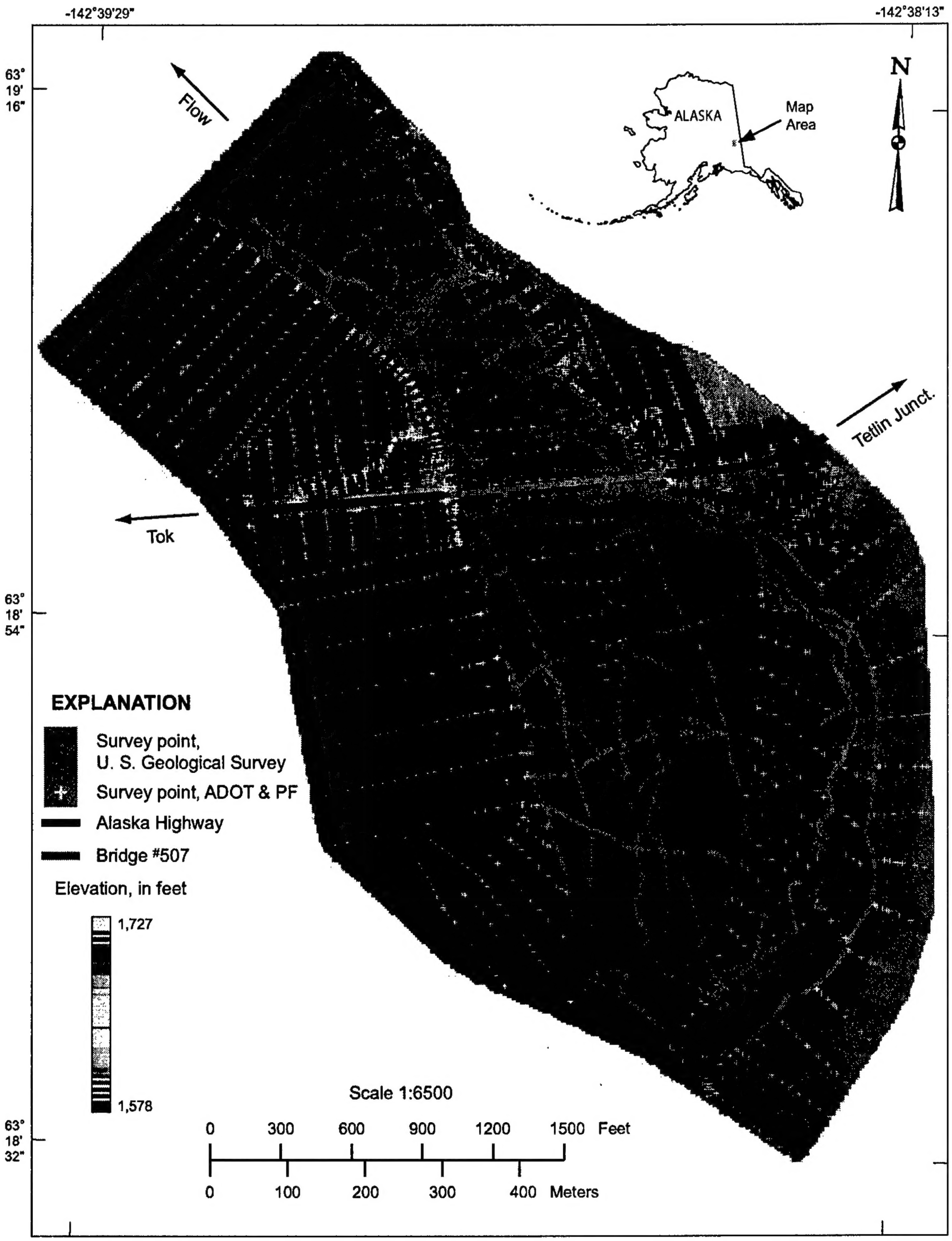

Figure 1. Digital elevation model of the Tanana River near Tok with data points collected by the USGS and ADOT&PF. Blue points are bathymetric data collected by the USGS and outline the extent of the channel. Yellow points are data collected above the water surface by ADOT&PF. Note: ADOT&PF data collected at a later date and lower stage than data collected by the USGS.

Background

The Tanana River near Tok drains an approximately 6,800 mi^2 glaciated basin north of the Alaska Range. At and near the study reach, the river is extensively braided and carries a high suspended-sediment load. At the bridge, the river occupies a single strand at high stages, but extensive bars develop during low flow and the channel continuously shifts. Two distinct thalwegs, one on the left bank and another on the right, are present in the vicinity of the bridge during high stages. The slope of the study reach is 0.0002 foot per foot (ft/ft). The riverbed is composed of silt and sand, and dune movement has been observed over a range of stages. The riverbanks are composed of unconsolidated silt and sand, with the exception of the right bank that is composed of granitic bedrock from the bridge to the downstream extent of the study reach. Overbank vegetation is predominately dense spruce and willow.

The existing bridge, ADOT&PF bridge number 505, is located at mile 80.5 of the Alaska Highway. It is 946 ft long, 23 ft wide, and was constructed in 1944. The bridge is a continuous steel through-truss with a laminated timber deck that is overlain by asphalt. Two sloping sharp-nosed piers that are surrounded by sheet pile support the bridge.

The U.S. Geological Survey (USGS) collected continuous daily streamflow information at the bridge (gaging station No. 15472000) from 1950 to 1953. The mean annual discharge of the Tanana River near Tok Junction during that period of data collection was 6,950 cubic feet per second (ft^3/s). During late May or early June, a period of high flow associated with snowmelt runoff begins. High flow persists typically through July and August reflecting glacial melt and periods of rainfall. High flows also result from intense rainfall in the late summer and low flows precede freeze up in late October to early November. Annual peak flows from 1950 to 1953 were 31,400, 31,400, 28,100, and 34,900 ft^3/s, respectively.

The bridge is located where the Tanana River enters a natural constriction that is controlled by the bedrock along the right bank. As the river approaches the bridge on the right bank, it is directed by the bedrock at an angle to the long axis of the pier. This flow angle of attack on the pier has been observed to be approximately 40 to 45 degrees at a discharge of 25,600 ft^3/s. The high angle of attack on the pier and position of the bridge at the head of a natural constriction, where flow velocities and sediment transport capacity are the highest, make this structure particularly susceptible to streambed scour.

Purpose and Scope

The purpose of this study was to provide ADOT&PF with bathymetric and hydraulic data to assist in the design and placement of a new bridge over the Tanana River at mile 80.5 of the Alaska Highway. This report documents two components: (1) the methodology for collection of the hydraulic and bathymetric data used to develop and calibrate a two-dimensional hydrodynamic model for a measured discharge of 25,600 ft^3/s and (2) the results of predictive numerical simulations using the calibrated model to evaluate hydraulic conditions for the 100-year recurrence interval discharge of 51,900 ft^3/s. This report presents the results from two simulations for the 100-year recurrence interval discharge: (1) modeling the existing channel conditions with the bridge piers and (2) modeling the discharge with the existing bridge removed to determine affects on flow velocity magnitude and directions.

Channel bathymetry and velocity magnitude and direction data were collected August 7-9, 2002 along an approximately 4,500 ft reach of the Tanana River that extended 2,000 ft downstream and 2,500 ft upstream of the bridge. A 1200-kHz acoustic Doppler current profiler (ADCP) and an echo sounder were interfaced with a Real-Time Kinematic Global Positioning System (RTK-GPS) to collect bathymetric and velocity vector data. Bathymetry data were combined with overbank data collected by ADOT&PF in October 2002. The combined data sets were used to construct a two-dimensional hydrodynamic model of an approximately 2,500 ft subsection of the study reach. The reach length was reduced to model the area around the bridge with greater resolution. The georeferenced velocity vectors collected with the ADCP and the RTK-GPS surveyed water-surface elevations were used to calibrate the hydrodynamic model.

Data Collection

Multi-dimensional models require significantly more topographic, bathymetric, and hydraulic data than do one-dimensional models. Recent advances in hydraulic and topographic surveying instrumentation have made this data collection more efficient and affordable. For this study, an RTK-GPS interfaced with an echo sounder allowed for the rapid collection of high-resolution and high-density data from a moving boat. Operating an ADCP synchronously with these instruments generated an extensive hydraulic data set for model calibration.

Data collection for the bathymetric survey spanned 3 days and the stage increased by 0.6 ft during this period (table 1). To account for the increasing stage, an average water-surface elevation and discharge were determined. Water-surface elevations surveyed before and after this time were adjusted to this average elevation to account for the rising stage. The water-surface elevation was surveyed daily at a stage reference location downstream from the bridge on the left bank.

All of the hydraulic and most of the bathymetric data were collected from a moving boat. Shallow areas of the channel were surveyed by personnel wading with the RTK-GPS. The RTK-GPS determined the horizontal and vertical position of each point collected by the echo sounder and ADCP. When in RTK mode, the GPS receives a position correction that is determined and broadcast by a separate GPS receiver set up over a survey-control station. The RTK-GPS made detailed measurements of water-surface elevations and the ADCP measured flow magnitude and direction. Simultaneously, channel-bathymetry data were being collected by a 200-kHz echo sounder.

Table 1. Location and elevations of survey reference points and stage reference point

[WSELEV, water-surface elevation]

Location (Alaska State Plane zone 2, NAD 83, NAVD 88 survey feet)			
Description	**Easting**	**Northing**	**Elevation**
ADOT&PF Control point 1	1533071.81	3404825.83	1,627.08
ADOT&PF Control point 2	1533711.52	3404928.48	1,625.14
ADOT&PF Control point 3	1534655.68	3405031.25	1,643.88
ADOT&PF Control point 4	1534942.47	3404923.68	1,672.00
08-07-02 Starting WSELEV	1533724.71	3405027.71	1,609.87
08-08-02 Starting WSELEV			1,610.09
08-08-02 closing WSELEV			1,610.24
08-09-02 Starting WSELEV			1,610.41
08-09-02 closing WSELEV			1,610.48

Position data that were collected using the RTK-GPS were referenced to position coordinates provided by ADOT&PF. Position data collected using the RTK-GPS have a manufacturer reported accuracy of ±0.033 ft ±1 parts per million (ppm) in the horizontal and ±0.066 ft ±1 ppm in the vertical while in RTK mode and ±0.016 ft ±0.5 ppm in the horizontal and ±0.016 ft ±1 ppm in the vertical for static survey mode (Trimble Navigation Limited, 2001). Measured accuracy of the echo sounder is ±0.1 ft.

Survey Control

ADOT&PF provided the position coordinates for four control points in the study reach (table 1). The horizontal control basis is Alaska State Plane zone 2 in feet, North American Datum of 1983 (NAD83), and the vertical control is based on the North American Vertical Datum of 1988 (NAVD88). The bathymetric survey was opened and closed each day by occupying one of the control points for 15 minutes at 15-second intervals with the RTK-GPS.

Bathymetric and Topographic Surveys

A 200-kHz echo sounder measured 19,270 channel depths over an approximately 5,000 ft reach of the river. Horizontal position and water-surface elevation were collected synchronously by the RTK-GPS. Hydrographic surveying software interfaced with both the RTK-GPS and echo sounder converted the channel soundings to elevations (Appendix 1). The channel bathymetry data were collected along cross sections, longitudinal profiles, and at random fill points (fig. 1).

ADOT&PF provided the topographic data of the areas outside of the channel including cultural features such as roads and the bridge and points collected on bars in the channel that were exposed at a stage lower than that at which the bathymetric data were collected. ADOT&PF data were collected in October 2002 at a stage of 1,604.2 ft (same stage reference location as the USGS survey) (fig. 1, Appendix 1).

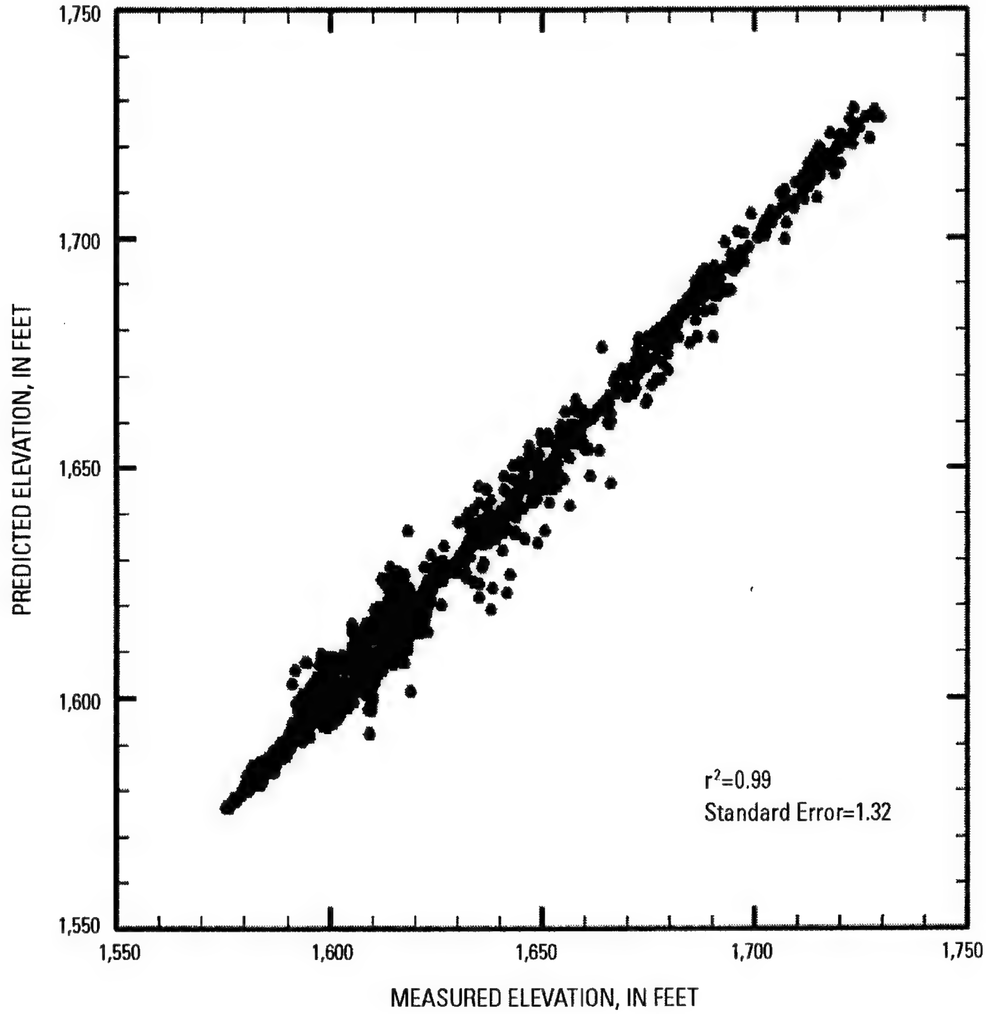

Figure 2. Predicted elevation values from statistical interpretation of topographic and bathymetric data plotted against measured values.

Surface Interpolation

The merged bathymetric and topographic datasets were of insufficient detail to develop an accurate and stable high resolution two-dimensional hydrodynamic model. Geostatistical software was used to interpolate elevation points from the available data using a technique known as spherical kriging. Kriging is an estimation procedure that computes values for unsampled areas based on the values of the points located around the unsampled area. A surface, such as topography, is a continuous surface and therefore a relationship exists from point to point. The degree of spatial dependence between sampled points along a specific orientation is known as the semivariance. Semivariance between measured points increases with the distance between those points up to a distance where the semivariance no longer increases. This distance defines the range of the regionalized variable where all points are related to one another. The range of the elevation data set collected for this study is 650 ft.

The elevation surface was interpolated from a topographic and bathymetric data set of 21,552 points using isotropic spherical-kriging in Gamma Design software (Gamma Design Software, 2001). The points were kriged with an active-lag distance of 820 ft, a lag class distance interval of 164 ft, and a search radius of 64 points at a distance of 1,640 ft. The interpolated surface includes 193,205 data points with an approximate 9 ft spacing between points. Pier elevations were manually input to the surface after the interpolation. Comparison of the predicted and measured values for elevation is shown in figure 2. The coefficient of determination (r^2) for this relationship is 0.99 and the standard error of the prediction is 1.32.

Hydraulic Data

Water-surface elevations were collected throughout the survey simultaneously with the bathymetric data using the RTK-GPS. Discrete RTK-GPS measurements of water-surface elevation at the stage reference point recorded a 0.6 ft increase in stage over the 3 days of data collection (table 1). A discharge measurement of 25,600 ft^3/s at a water-surface elevation of 1,610.2 ft made on August 8, 2002, with the ADCP is considered an average discharge for the survey. The water-surface elevation was measured at the stage reference location and also was considered an average for the survey. Elevations collected before and after this were adjusted to reflect the rising stage.

The ADCP measures three-dimensional velocities from approximately 1 ft below the water surface to within 6 percent of the depth to the bottom. Velocities were measured every 0.82 ft within this section of the water column. The unmeasured sections of the water column are estimated by the ADCP software using established methods (Simpson, 2001). These data were collected throughout the survey and along pre-determined cross sections for use in model calibration. Depth-averaged velocities and flow directions were then computed from the ADCP cross sections. These averages only include data from within the area of the water column that the ADCP can measure. Analysis of depth-averaged ADCP velocities by Wagner and Mueller (2001) showed that these values should be within 5 percent of the mean if the entire water column would have been measured. These data are used for the comparison of modeled flow velocities and calibration of the eddy viscosity parameter and are discussed below.

The magnitude of the 100-year recurrence interval discharge, 51,900 ft^3/s, was determined for a statewide assessment of streambed scour at bridges by Heinrichs and others (2001). Heinrichs and others (2001) used regional regression equations developed by Jones and Fahl (1994) to determine the magnitude of the recurrence interval floods at select bridges in Alaska. The regression equations used statistically significant basin characteristics including basin area, mean basin elevation, mean annual precipitation, mean minimum January temperature, and percentage of basin covered by forest or covered by lakes or ponds.

Hydrodynamic Model

The USGS Multi-Dimensional Surface Water Modeling System (MD_SWMS) (R.R. McDonald, U.S. Geological Survey, oral commun., 2003, (http://wwwbrr.cr.usgs.gov/projects/SW_Math_mod/OpModels/MD_SWMS/index.htm) was selected to calculate water-surface elevation, flow velocity, and flow direction throughout a length of approximately 2,500 ft of the Tanana River in the vicinity of the bridge. A two-dimensional hydrodynamic model was chosen over a one-dimensional model for two reasons. First, reverse flow was observed at several locations in the study reach at the calibration discharge and a one-dimensional flow model assumes unidirectional flow. Second, one-dimensional models do not compute horizontal flow directions, which are critical for scour analyses and the design of bridge piers.

MD_SWMS is a generic Graphical User Interface (GUI) reading and writing to a data structure that is general enough to be used by any numerical model of flow and transport. Currently, the interface uses a two-dimensional, vertically-averaged steady-state flow model with a curvilinear orthogonal or simple rectilinear grid. The model calculates water-surface elevations and two-dimensional horizontal velocity components for each wetted cell in the user-defined grid. Velocities in the vertical direction are not computed. The numerical basis of the model is described in detail by Nelson and others (2003).

Mesh Configuration

Curvilinear orthogonal grids were created in MD_SWMS about a center line in the channel. The model of the 100-year discharge required a separate wider grid to incorporate flow that is out of bank. The initial grid consists of 88,688 cells spaced every 6.5 ft in both the downstream and cross-stream directions. Grid dimensions are 2,523 ft in the downstream direction by 1,652 ft in the cross-stream direction. The mesh created for the 100-year discharge consists of 120,744 cells spaced every 6.5 ft for 2,257 ft in the downstream direction and 2,303 ft in the cross-stream direction.

Boundary Conditions and Model Calibration

Initial boundary conditions for simulations of the calibration discharge and 100-year discharge included a starting downstream water-surface elevation, water-surface slope, and discharge. The drag coefficient and lateral eddy viscosity were model parameters that were validated with field data from the calibration discharge and then used for the simulation of the 100-year discharge. The drag coefficient was used to evaluate frictional losses associated with channel roughness and the lateral eddy viscosity was a coefficient used to approximate lateral turbulent diffusion or lateral momentum transfer. Boundary conditions and model parameters are summarized in table 2 and discussed below.

Model calibration was achieved through an iterative process of varying drag coefficients until values computed by the model for water-surface elevation, velocity, and flow directions reproduced as closely as possible the measured values. Graphical tools and statistics were used to compare predicted and measured values.

This model was calibrated for a steady-state discharge of 25,600 ft^3/s and represents channel and hydraulic conditions at the time of the survey. The downstream boundary condition for this discharge was an averaged surveyed water-surface elevation of 1,610.14 ft. The upstream boundary condition was a surveyed water-surface slope of 0.0002 ft/ft.

An initial drag coefficient was assigned to each of the grid cells based on bed material type and, in the case of the overbanks, the type and density of the vegetation. The drag coefficient was iteratively adjusted until the modeled water-surface elevations and velocities generally agreed with measured values for the given discharge. The selected drag coefficients resulted in the closest approximation of water-surface elevations (fig. 3) and flow velocities (fig. 4). An increase in the drag coefficient resulted in higher modeled water-surface elevations and lower flow velocities. The opposite was true of a decrease in the drag coefficient. The dimensionless drag coefficient for the channel is 0.004 and 0.12 for the overbanks. An equivalent Manning's *n* value can be found using a depth scale, here the mean depth in feet, with equation 1:

$$n_{Manning's} = 1.515\sqrt{C_d\left(\frac{H^{\frac{1}{3}}}{g}\right)} \quad (1)$$

where

$n_{Manning's}$ is the equivalent Manning's roughness value,

C_d is the dimensionless drag coefficient,

H is the mean depth of flow, in feet, and

g is the constant for acceleration due to gravity, in feet squared per second.

Using a mean depth of 7.2 ft for the calibration flow and 11.5 ft for the 100-year discharge the equivalent Manning's n values for the channel are 0.023 and 0.025, respectively. For one-dimensional models, the roughness value typically decreases with increasing flow depth, but the drag coefficient used in multi-dimensional models is not depth dependent. It represents the roughness of the entire channel, including the banks. The equivalent Manning's n computed using equation 1 must therefore increase with an increase in flow depth while the drag coefficient is constant for any depth of flow. The drag coefficient assigned to the overbanks was 0.12, which is roughly equivalent to a Manning's roughness value of 0.09 for a flow depth of 0.72 ft.

The lateral eddy viscosity was initially estimated as a coefficient (0.01) times both the mean depth in feet (approximately 7.2 ft) and mean velocity (approximately 2.9 ft/s). This initial eddy viscosity was then iteratively increased to a value of 4.3 ft^2/s; this is the lowest value that produced a numerically stable simulation of the discharge. Increasing the lateral eddy viscosity increased the stability of the solution, but also decreased the two-dimensional variability in the modeled flow. Plotting averaged ADCP flow vectors against modeled flow vectors helps evaluate if the lateral eddy viscosity selected represents observed conditions (fig. 5). Comparison of predicted and measured values for hydraulics was complicated by the fact that the measured velocities and directions are collected at a point in space moving continuously across a river over a short time interval while modeled values represented a true average of these values. A greater variability in both magnitude and direction can be expected in the measured velocities. Results from the simulation of the calibration discharge are presented in Appendix 2.

Table 2. Boundary conditions and model parameters used for the simulations of the calibration discharge of 25,600 ft^3/s and 100-year recurrence interval discharge

[ft^3/s, cubic feet per second; ft, foot; ft/ft, foot per foot; ft^2/s, feet squared per second]

Boundary conditions	Calibration discharge	100-year discharge
Discharge	25,600 ft^3/s	51,900 ft^3/s
Downstream starting water-surface elevation	1,610.14 ft	1,613.41 ft
Slope	0.0002 ft/ft	0.0002 ft/ft
Model parameters		
Number of grid cells	61,908	120,744
Grid cell spacing in stream-wise and stream-normal directions	6.5 ft	6.5 ft
Channel drag coefficient	0.004	0.004
Overbank drag coefficient	0.12	0.12
Lateral eddy viscosity	4.3 ft^2/s	4.3 ft^2/s

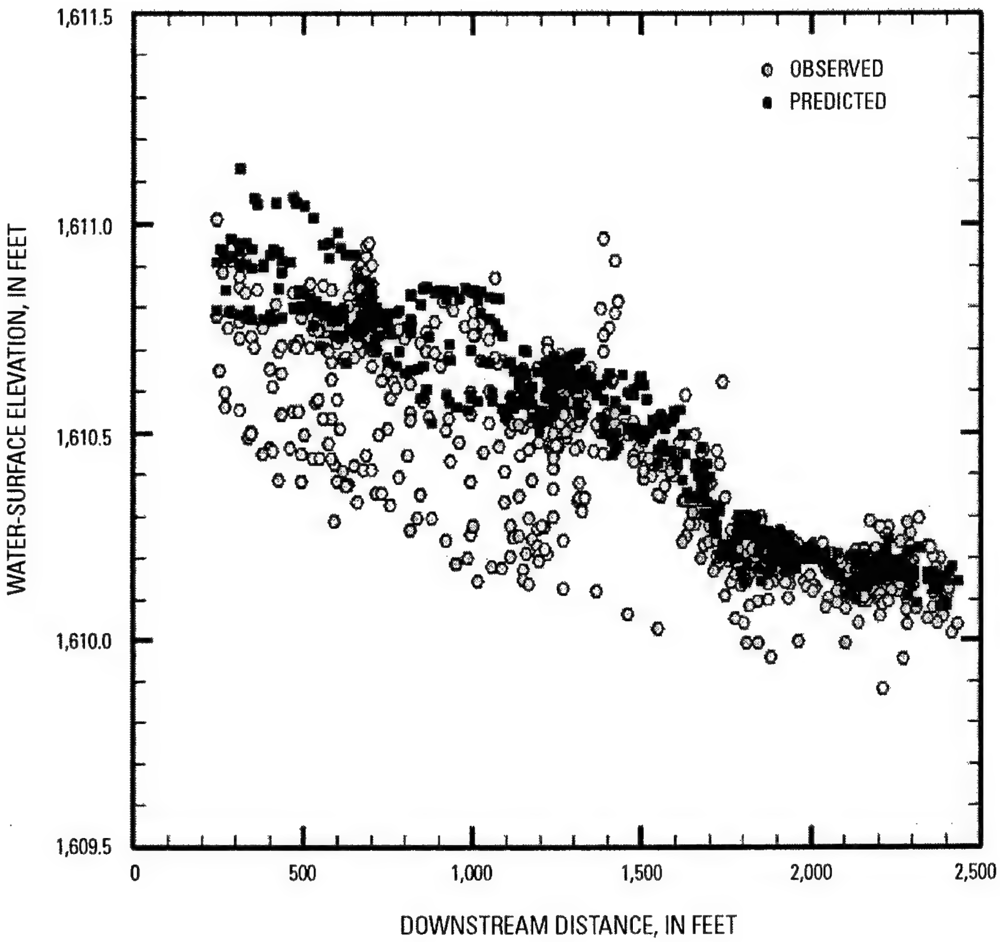

Figure 3. Predicted and observed values for water-surface elevation for the calibration discharge of 25,600 ft^3/s at bridge 505 of the Alaska Highway over the Tanana River near Tok, Alaska.

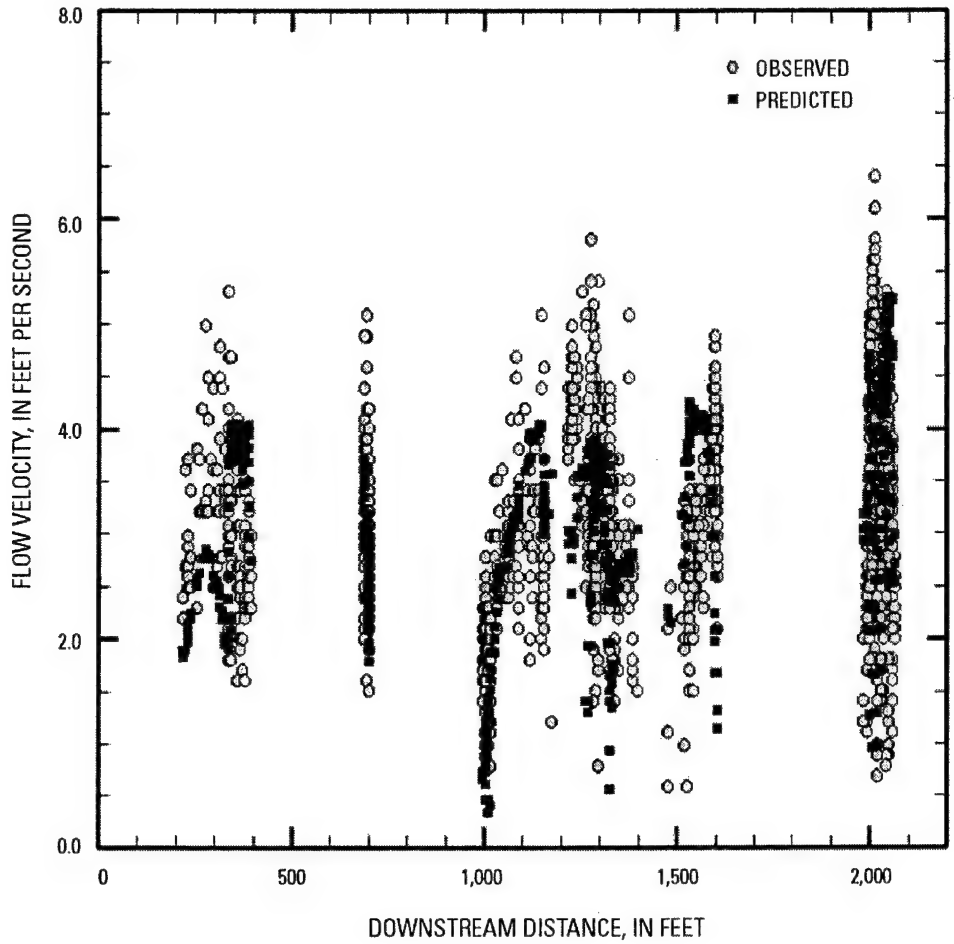

Figure 4. Predicted and observed values for water velocity for the calibration discharge of 25,600 ft^3/s at bridge 505 of the Alaska Highway over the Tanana River near Tok, Alaska.
The distribution of points in the vertical direction represents the variation in velocity magnitude across the channel.

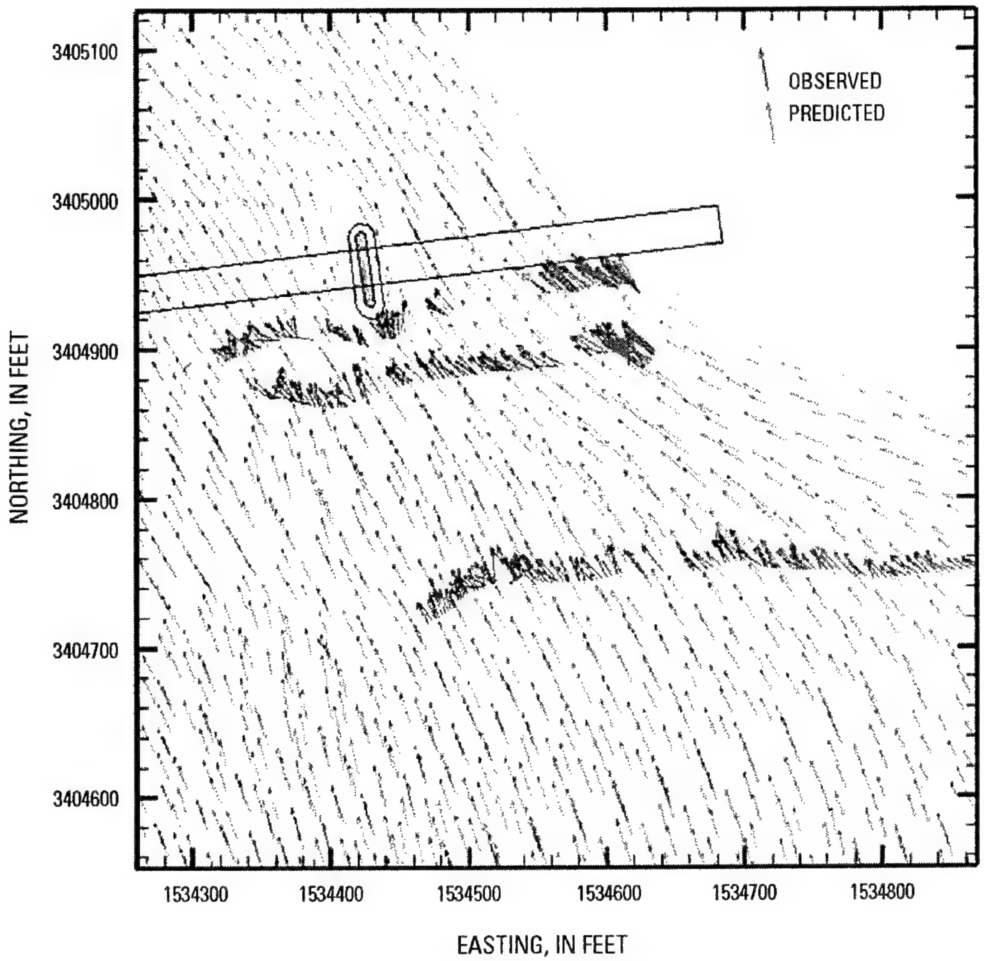

Figure 5. Predicted and observed velocity vectors for the calibration discharge of 25,600 ft^3/s at bridge 505 of the Alaska Highway over the Tanana River near Tok, Alaska.

The downstream boundary condition for the simulation of the 100-year recurrence interval discharge was an initial water-surface elevation, which was computed using a one-dimensional solution. Cross-sectional geometry from the interpolated surface was input into the U.S. Army Corps of Engineers' Hydrologic Engineering Center River Analysis System (HEC-RAS) (Brunner, 2002) using a slope of 0.0002 ft/ft as a downstream boundary condition and the roughness values discussed above. The one-dimensional solution for an average-water surface at the downstream cross section was 1,613.41 ft for the 100-year recurrence interval discharge of 51,900 ft^3/s. This elevation was the downstream boundary condition and the water-surface slope of 0.0002 ft/ft was the upstream boundary for the two-dimensional simulation. With these boundary conditions and the model parameters from the calibration discharge, a stable two-dimensional solution was achieved.

Model convergence was evaluated by comparing the predicted model discharge to the specified discharge for a cross section. After 200 iterations, the percentage of deviation from the normalized discharge at each cross section was within ±3 percent for the simulations of the calibration discharge and 100-year recurrence interval discharge. Acceptable values are generally thought to be ±3 percent (R.R. McDonald, U.S. Geological Survey, oral commun., 2003).

Results from the 100-year Recurrence Interval Discharge Model

Coordinates, elevations, modeled water-surface elevations, flow velocities, and horizontal velocity components for this simulation are presented in Appendix 3. The modeled water surface was contained within the banks at and downstream of the bridge. Overbank flow only occurred along the right bank upstream of the bridge. The modeled flow extended to the edge of the grid along a small portion of the right bank where flow depths were less than 1 ft and flow velocity approached 0 ft/s (fig. 6).

Predicted velocities were as high as 8.7 ft/s along the left bank at the bridge and downstream from this point. Predicted velocities at the bridge piers were low because the model bathymetry includes large scour holes that were present during the field survey. The predicted flow angle of attack on both piers was higher than the modeled angles for the calibration discharge. The predicted angles of attack for the left and right bank piers were 45 and 55 degrees respectively, an increase of 7 degrees for the left bank pier and 15 degrees for the right bank pier from the calibrated discharge (fig. 7).

A second simulation of the 100-year recurrence interval discharge was made using a new channel geometry that did not include the existing bridge piers. The results from this simulation illustrate the extensive downstream effects of the bridge piers on both flow velocity and direction (fig. 7). Results from this simulation also can be used for a bridge design scenario that includes removing the existing structure. Removing the existing bridge piers from the model resulted in an increase in flow velocity magnitude in the areas that were in the downstream shadow of the piers, but overall, velocity across the channel decreased for this scenario. Flow directions from this scenario can be used to determine channel flow angle of attack on piers for the replacement bridge. Coordinates, elevations, water-surface elevations, flow velocities, and the horizontal velocity components for this simulation are presented in Appendix 4.

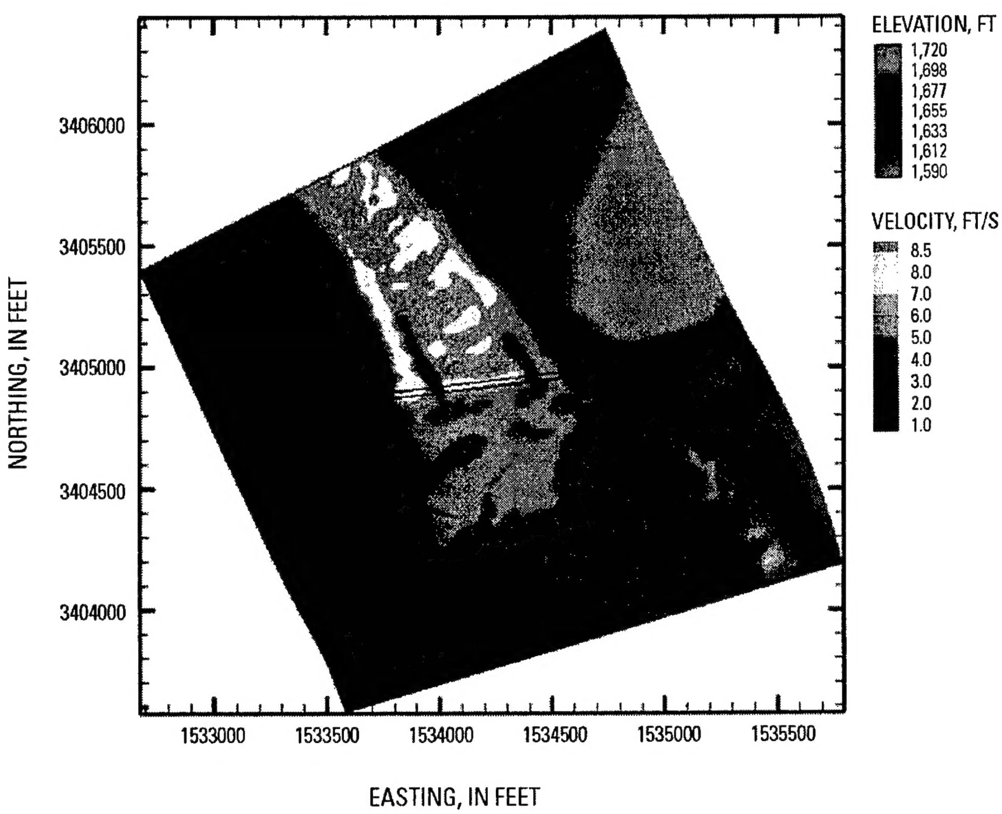

Figure 6. Predicted flow velocity and extent of water surface for the simulation of the 100-year recurrence interval discharge of 51,900 ft^3/s at bridge 505 of the Alaska Highway over the Tanana River near Tok, Alaska.

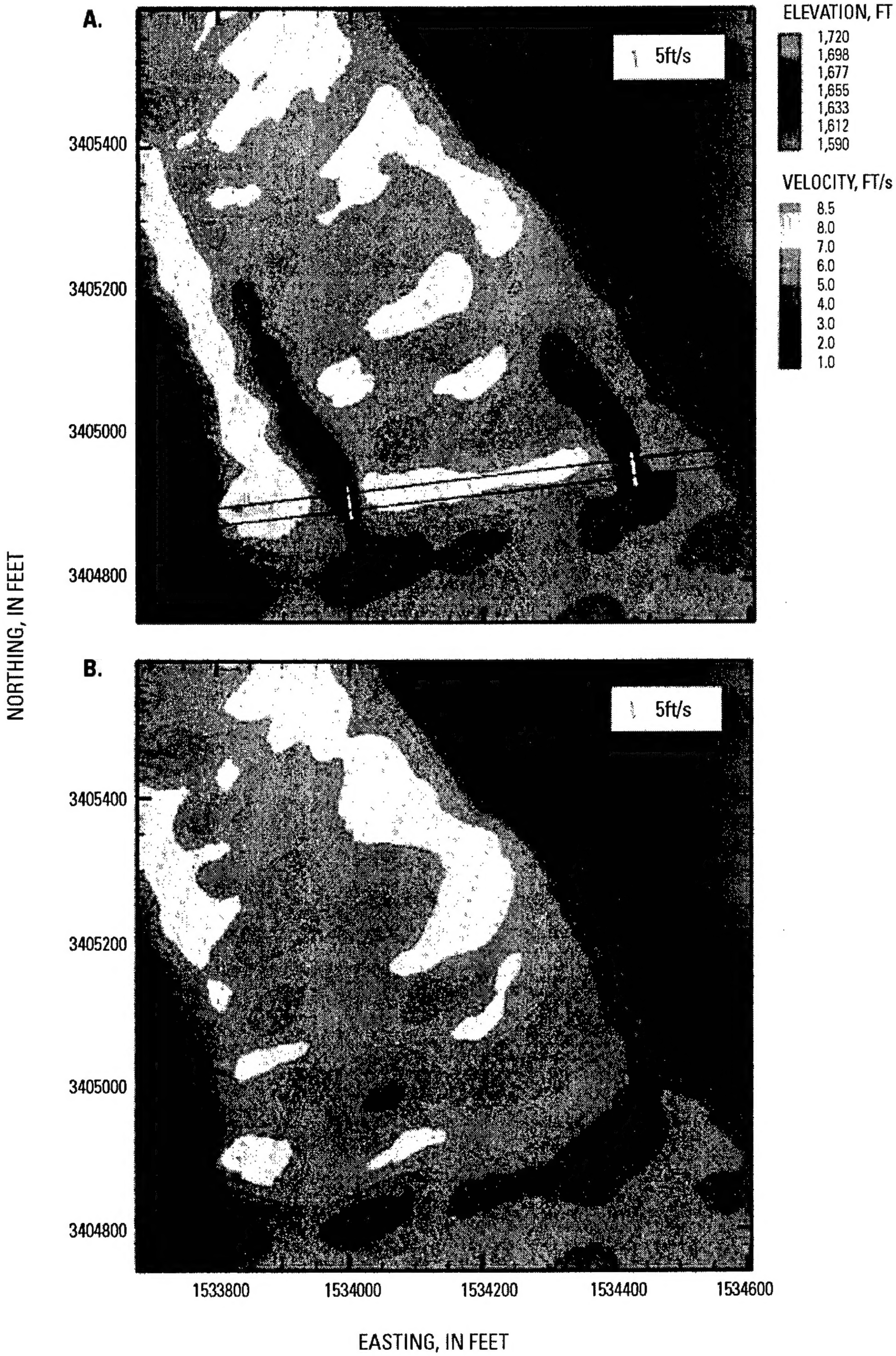

Figure 7. A. Velocity vectors and contours of velocity for the simulation of the 100-year recurrence interval discharge including the existing bridge piers. B. Velocity vectors and contours of velocity for the same discharge with the existing piers removed.

Summary

This report presents the methods used, data collected, and hydrodynamic model results from the USGS and ADOT&PF study to collect bathymetric and topographic data for a hydrodynamic model of the Tanana River where bridge 505 of the Alaska Highway makes its crossing. The USGS collected an extensive bathymetric and hydraulic data set in the vicinity of the bridge using ADCP, RTK-GPS, and an echo sounder. These data, along with survey data of the overbanks collected by ADOT&PF were used to construct a two-dimensional hydrodynamic model in the vicinity of the bridge.

A calibration discharge of 25,600 ft^3/s was simulated with the U.S. Geological Survey's Multi-Dimensional Surface Water Modeling System. This model is a generic interface that currently uses a two-dimensional, vertically averaged steady-state flow model with curvilinear orthogonal or simple rectilinear grid. Boundary conditions and model parameters were calibrated to data collected during the hydraulic survey. The drag coefficient for the channel was calibrated through an iterative process where predicted water-surface elevations and flow velocity magnitudes were compared to measured values. The lateral eddy viscosity was also calibrated though an iterative process and predicted velocity vectors compared to those measured by the ADCP.

A model of the 100-year recurrence interval discharge was constructed using calibrated parameters from the simulation of the measured discharge and a starting water-surface elevation calculated with a one-dimensional model. Predicted flow angle of attacks increased from 38 to 45 degrees and 45 to 55 degrees on the left- and right-bank piers, respectively. A second simulation of this discharge was run with the existing piers removed from the channel. Predicted velocities, water-surface elevations, and velocity vectors from these simulations will aid ADOT&PF in the design of a new bridge in the event that the existing bridge is removed from the channel.

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Manuscript approved for publication, April 28, 2004

Prepared by the Publishing Group, U.S. Geological Survey,
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